Seismic fragility analyses of a CANDU 6 reactor building nuclear power plant

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SUMMARY:
Within the scope of the refurbishment project of a CANDU 6 nuclear power plant (NPP), a new seismic demand characterised by a uniform hazard spectra (UHS) is obtained for a return period of 1/10 000 years. In this context, seismic fragility analysis is an important step in the evaluation of the seismic margin with regard to the capacity of components (structures) of the plant to sustain the new seismic demand. In this paper, the methodology adopted for the seismic fragility analyses of this CANDU 6 reactor building is presented. The fragility analyses are mainly based on the Electric Power Research Institute (EPRI) approach combined with structural models calibrated with ambient vibrations measurements results. The results are presented for the containment wall and for the internal structure. The fragility analyses methodology adopted for this CANDU 6 NPP is particularly useful to engineers involved in fragility analyses of complex structural systems.

Keywords: CANDU 6 NPP, Seismic fragility, Reactor building.

1. INTRODUCTION

Within the scope of the refurbishment project of a CANDU 6 nuclear power plant (NPP), a new seismic demand is obtained for the site. This new seismic demand is characterised by a uniform hazard spectra (UHS), and derived from a site specific study for a return period of 1/10 000 years (Hydro Quebec, 2009-a). Compared to the original seismic design demand based on Newmark-Housner type of ground response spectra, the UHS for the site of study exhibits larger spectral ordinates in the high frequency range.

However, the safety of this NPP with regard to earthquakes should account for the uncertainties associated with the occurrence of seismic events and their effects on the components (structures) of the plant. This motivates the use of a probabilistic approach for the seismic safety assessment. For a NPP in operation, the probabilistic approach allows the evaluation of the current safety considering on a rational basis the aleatory and epistemic uncertainties as well as the regulator requirements. Within the probabilistic seismic safety assessment, seismic fragility analysis is an important step in the evaluation of the seismic margin with regard to the capacity of components (structures) of the plant to sustain the new seismic demand. The seismic fragility of the components (structures) can be defined by the conditional probability of failure for a given seismic parameter. For the probabilistic seismic safety assessment of the plant, the resulting performance index from the seismic fragility analysis is the High Confidence of Low Probability of Failure (HCLPF) seismic capacity, defined in terms of the selected seismic parameter.

In this paper, the methodology adopted for the seismic fragility analyses of a CANDU 6 reactor building is presented (Hydro-Quebec, 2010-b). The fragility analyses are mainly based on the Electric Power Research Institute (EPRI) approach (1991; 1994; 2002) combined with structural models calibrated with ambient vibrations measurements results (Nour et al., 2010; Hydro-Quebec, 2009-b). The results are presented for the containment wall (CW) and for the internal structure (IS).

The fragility analyses methodology adopted for this CANDU 6 NPP is particularly useful to engineers involved in fragility analyses of complex structural systems.

2. METHODOLOGY FOR THE SEISMIC FRAGILITY EVALUATION FOR STRUCTURES

2.1 Seismic fragility curves

The seismic fragility of a structure can be defined by the conditional probability of failure for a given seismic parameter, i.e, the peak ground acceleration (PGA) or the spectral ordinate around the fundamental frequency of the structure. The capacity estimation in terms of the chosen seismic parameter is generally obtained from the available information in the design basis. This includes the geometry of the structure, material properties, and the structural response considering the seismic design data. There are several sources of randomness and epistemic uncertainty that affect the accurate estimation of the structural capacity of each potential failure mode. These sources of uncertainty can indeed affect significantly the structural capacity expressed in sustainable acceleration of the structure. Therefore, the seismic fragility is usually described by a family of curves associated with a predefined probability value to reflect the level of confidence in the estimation of the fragility. The fragility *A* of the structure corresponding to a particular failure mode can be expressed in terms of the median ground acceleration capacity A_M and the two random variables ε_R and ε_U as follows (Kennedy et al., 1980):

$$A = A_M \cdot \varepsilon_R \cdot \varepsilon_U \tag{2.1}$$

Here, ε_R and ε_U are random variables having median values equal to unity. They represent the inherent randomness around the median value and the epistemic uncertainty in the median capacity value. They are supposed to have a lognormal distribution with logarithmic standard deviations β_R and β_U respectively. The uncertainty in the evaluation of the fragility is usually expressed in terms of family values of the probability of failure for a given value of ground acceleration (or the chosen seismic parameter). Thus, the probability *p* that the conditional probability of failure p_f exceeds a specified value p_f', for a given ground acceleration value "*a*" (the chosen seismic parameter), is given by (Kennedy et al., 1980):

$$p\left[\frac{p_f > p_f'}{a}\right] = \phi\left[\frac{\ln\left\{\left(\frac{a}{A_M}\right)\cdot \exp\left[\beta_R \cdot \phi^{-1}\left(p_f'\right)\right]\right\}}{\beta_U}\right] \tag{2.2}$$

The conditional probability of failure p'_f for a no-exceedance probability *Q* can be expressed as follows (Kennedy et al., 1980):

$$p_f' = \phi\left[\frac{\ln\left(\frac{a}{A_M}\right) + \beta_U \cdot \phi^{-1}(Q)}{\beta_R}\right] \quad ; \quad Q = 1 - P \tag{2.3}$$

$\phi(x)$ is the cumulative function of the standard Gaussian distribution; $\phi^{-1}(x)$ denotes the inverse function, *Q* is the probability of no-exceeding, in practice the values of 5%, 50% and 95% are often used. Note that the mean curve is defined in terms of the composite variable β_C as follows (EPRI, 1994):

$$p_0 = \phi\left[\frac{\ln\left(\frac{a}{A_M}\right)}{\beta_C}\right] \quad ; \text{ with } \quad \beta_C = \sqrt{\beta_R^2 + \beta_U^2} \tag{2.4}$$

The mean curve represents a best estimate of the fragility curves without separating explicitly the randomness from the epistemic uncertainty.

2.2 High Confidence of Low Probability of Failure (HCLPF) for structures

Following the methodology described in EPRI (1994), and used for the seismic risk assessment for more than 50 NPP in the United States, the high confidence (95%) for a low Probability of Failure (5%) is defined by the following equation:

$$HCLPF = A_M \cdot \exp[-1.65 \cdot (\beta_R + \beta_U)] \tag{2.5}$$

The median ground motion capacity is defined by:

$$A_M = F_M \cdot A_{UHS} \tag{2.6}$$

Where F_M is the median safety factor and A_{UHS} represents the median spectral ordinate of the median UHS 2008 (Hydro-Quebec, 2009-a). A_{UHS} is determined around the reference period $T_1 = 1/(7.4\,\text{Hz})$ as recommended by the Seismic Design Guide for this CANDU 6 NPP (Hydro-Quebec, 2010-a):

$$A_{UHS} = Sa(T_1) = \frac{1}{(T_1^+ - T_1^-)} \int_{-T}^{+T} Sa(T) \cdot dT \tag{2.7}$$

To reflect the uncertainty over the reference period, a ± 15% variation is considered, i.e., $T_1^- = 0.85 \cdot T_1$ and $T_1^+ = 1.15 \cdot T_1$. It is now clear that the chosen seismic parameter for the seismic fragility analysis is A_{UHS}, because as indicated in the EPRI documents (1994 and 2002), the use of the PGA introduces additional uncertainties in the analysis. Moreover, the structural frequencies of interest are in most cases below 10 Hz, i.e., too far from the frequency at which the PGA is defined (around 100 Hz in our case). This allows a more accurate seismic risk determination.

The scope of the seismic fragility analysis is to evaluate the seismic margins in the structural response by examining the data used in the design and their comparison with the current reality of the structure. In other words, it is necessary to eliminate conservatism to find the median seismic capacity. For structures, the median safety factor F_M is defined as (EPRI, 1994):

$$F_M = F_C \cdot F_{SR} \tag{2.8}$$

F_C represents the seismic capacity factor and F_{SR} is the structural response factor. F_C is expressed as follows:

$$F_C = F_S \cdot F_\mu \tag{2.9}$$

F_μ : is the inelastic energy absorption factor. It is determined by estimating the de-amplification of post-elastic structural response due to the ductility combined with the mobilised damping in the facility.
F_S : is the safety factor. It represents the relationship between the ultimate resistance, for which there is loss of functionality of the structural element, and the current resistance. This factor is defined as follows:

$$F_S = \frac{S - P_N}{P_{UHS}} \quad \text{with} \quad P_{UHS} = P_T - P_N \tag{2.10}$$

where :

S : describes the resistance of the structural element for a well specified failure mode.
P_N : denotes the normal operating load "NOL" (dead loads, operating loads, etc...).
P_T : is the total load supported by the structure, which is the sum of the seismic loads obtained from the median UHS 2008 and the " NOL" loads.
P_{UHS} : represents the seismic load obtained from the median UHS 2008.

The structural response factor F_{SR} is defined as the product of all factors influencing the variability of the structural response. It is expressed as follows:

$$F_{SR} = F_{SA} \cdot F_{GMI} \cdot F_\delta \cdot F_{Md} \cdot F_{MC} \cdot F_{EC} \cdot F_{SSI} \tag{2.11}$$

where :

F_{SA} : is the seismic motion factor. This factor takes into account the spectral shape, the horizontal PGA and the vertical seismic component.
F_{GMI} : is the surface ground motion wave incoherency factor.
F_δ : is the damping factor. It represents the variability of the structural response due to the difference between the current damping and the one used in the design.
F_{Md} : is the factor which accounts for the structural modeling. It takes into account the uncertainties of the structural response with respect to the modeling assumptions.
F_{MC} : is the factor that accounts for the modal combination of different modes of vibration.
F_{EC} : is the factor that considers the combination of structural responses due to different earthquake components.
F_{SSI} : is the soil-structure interaction factor.

The randomness β_R and the uncertainty β_U associated with the median safety factor are defined as follows (EPRI, 2002):

$$\begin{gathered}
\beta_R = \sqrt{\beta_{R_C}{}^2 + \beta_{R_SR}{}^2} \\
\beta_R = \sqrt{\left(\beta_{R_S}{}^2 + \beta_{R_\mu}{}^2\right) + \left(\beta_{R_SA}{}^2 + \beta_{R_GMI}{}^2 + \beta_{R_\delta}{}^2 + \beta_{R_Md}{}^2 + \beta_{R_MC}{}^2 + \beta_{R_EC}{}^2 + \beta_{R_SSI}{}^2\right)} \\
\beta_U = \sqrt{\beta_{U_C}{}^2 + \beta_{U_SR}{}^2} \\
\beta_U = \sqrt{\left(\beta_{U_S}{}^2 + \beta_{U_\mu}{}^2\right) + \left(\beta_{U_SA}{}^2 + \beta_{U_GMI}{}^2 + \beta_{U_\delta}{}^2 + \beta_{U_Md}{}^2 + \beta_{U_MC}{}^2 + \beta_{U_EC}{}^2 + \beta_{U_SSI}{}^2\right)}
\end{gathered} \tag{2.12}$$

where :

β_{R_C}, β_{U_C} : randomness and uncertainty associated with the seismic capacity factor.
β_{R_SR}, β_{U_SR} : randomness and uncertainty associated with the structural response factor.
β_{R_S}, β_{U_S} : randomness and uncertainty associated with the strength factor.
β_{R_μ}, β_{U_μ} : randomness and uncertainty associated with the inelastic energy absorption factor.
β_{R_SA}, β_{U_SA} : randomness and uncertainty associated with the seismic ground factor.
β_{R_δ}, β_{U_δ} : randomness and uncertainty associated with the damping factor.
β_{R_Md}, β_{U_Md} : randomness and uncertainty associated with the structural modeling factor.
β_{R_MC}, β_{U_MC} : randomness and uncertainty associated with the modal combination factor.
β_{R_GMI}, β_{U_GMI} : randomness and uncertainty associated with the seismic wave incoherency factor.
β_{R_EC}, β_{U_EC} : randomness and uncertainty associated with different earthquake components combination factor.
β_{R_SSI}, β_{U_SSI} : randomness and uncertainty associated with the soil-structure interaction factor.

The median equations must be used to determine the structural element capacity for a specified failure mode. According to the ASCE 43-05, the ultimate strength equations recommended by building codes are in most cases identified with a conservative and have at least 98% probability of exceedance. To make the code equations median (50%), ASCE 43-05 recommends to correct the R_s factor, defined as a function of the uncertainty associated with resistance factor β_{U_S}, as follows:

- for ductile elements :

$$R_s = \exp(2.054 \cdot \beta_{U\_S}) \qquad (2.13)$$

- for low ductile elements :

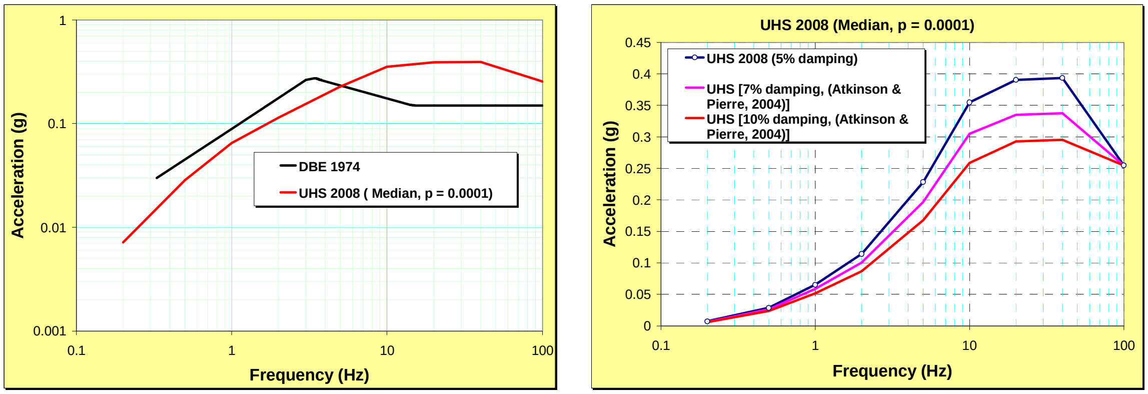
$$R_s = 1.33 \cdot \exp(2.054 \cdot \beta_{U\_S}) \qquad (2.14)$$

2.3 Median seismic motion

As recommended by EPRI (1994, 2002), seismic fragility analysis for structures must be conducted using median seismic inputs. For our case, a site specific study was conducted by Atkinson (Hydro-Quebec, 2009-a). Compared to the seismic design data (DBE 74), the updated study defines the seismic action by means of a median UHS 2008. As shown by Figure 1.a, the median UHS 2008 makes in evidence a significant increase of spectral ordinates at high frequencies.

The energy dissipation in the structures, to response levels close to the material elastic limit, is assumed generally to depend on the velocity which is similar to the behavior of a viscous damper. In fact, the damping is estimated from the observations and is considered to depend on the strain level. In this context, EPRI (1994) recommends values for material damping depending on the stress state that is half of the elastic limit, near or above the elastic limit. For seismic fragility analyses, the damping values to a stress level close to the elastic limit should be used.

The median UHS 2008 was developed for a damping coefficient of 5%. For other damping levels, it is recommended to use the method of Atkinson and Pierre (2004) which is very appropriate for sites in Eastern North America (ENA). In this case, as shown in Figure 1.b, the 5% spectrum is modified by coefficients that depend on the frequency (Atkinson and Pierre, 2004).



a. b.

Figure 1. Uniform hazard spectra UHS 2008 for the site of study.

3. SEISMIC EVALUATOIN OF THE REACTOR BUILDING CAPACITY

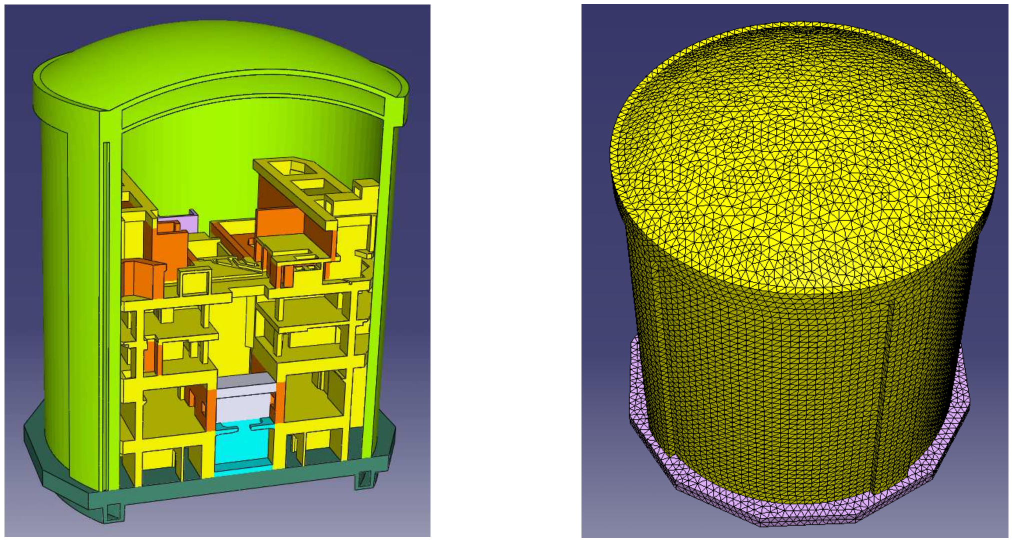
For the reactor building seismic fragility analyses, the seismic demand is computed from a 3D finite element model developed using the ABAQUS computer code. As shown in Figure 2, this model includes both, the containment wall, and the internal structure. Linear seismic analyses were conducted using the modal spectral method and considering a composite material damping, i.e., 3% for the prestressed concrete structure and 5% for the reinforced concrete structure. Moreover, the 5% damping median UHS 2008 was selected as the design basis earthquake for the calculation of the linear elastic seismic demand of the containment wall and the internal structure.

Exhaustive details of the 3D finite element model, ambient vibrations measurements, the calibration of the numerical model with the ambient vibrations measurements and the used material properties are presented in Nour et al. (2010); Hydro Quebec (2009-b) and IZIIS (2009).

Because the material damping of the containment wall is taken equal to 3%, this leads in one hand, to an underestimation of the seismic demand, and in the other hand, to an overestimation of the safety factor by an amount equal to the ratio of amplification factors (AF), i.e., $AF_{3\%}/AF_{5\%}$. To be coherent with our calculations, this overestimation is balanced by the reduction of the structural response factor by the same factor $AF_{3\%}/AF_{5\%}$ (more specifically by reducing the damping factor F_{δ}).

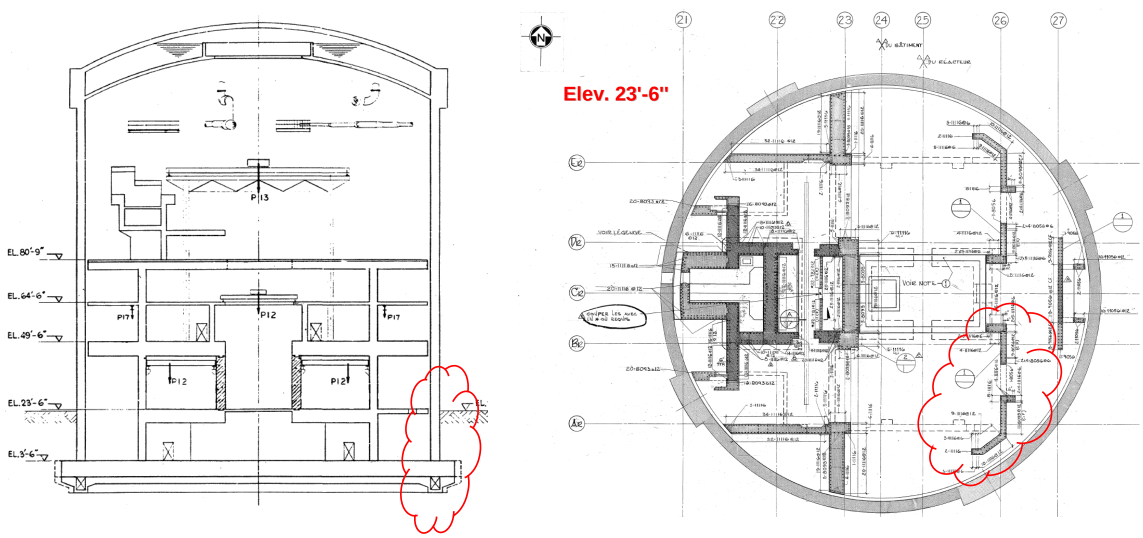
For structures similar to the containment wall, the recent studies conducted on the other CANDU NPP (Park et al., 1998; Lee and Song, 1999; Choi et al., 2008) showed that the most critical failure mode corresponds to the tangential shear at the base. It is found that this failure mode is governing as well, for the containment wall of the CANDU 6 reactor building (see Figure 3.a). The capacity in terms of HCLPF for this failure mode represents the lowest value among all potential failure modes. It is worthy to mention that the containment wall is locally reinforced by adding steel reinforcements and by increasing the wall thickness at critical penetrations and airlocks. Therefore, the failure in these areas will not govern.

The calculation details of the high confidence of low probability of failure (HCLPF) for the containment wall are presented in Hydro-Quebec (2010-b). However, the synthesis of the results is given in Table 3.1, and the fragility curves corresponding to 95%, median (50%), mean and 5% confidence levels are shown in Figure 4.a.



a. b.

Figure 2. Finite element model of the CANDU 6 reactor building.



a. Containment wall b. Internal structure

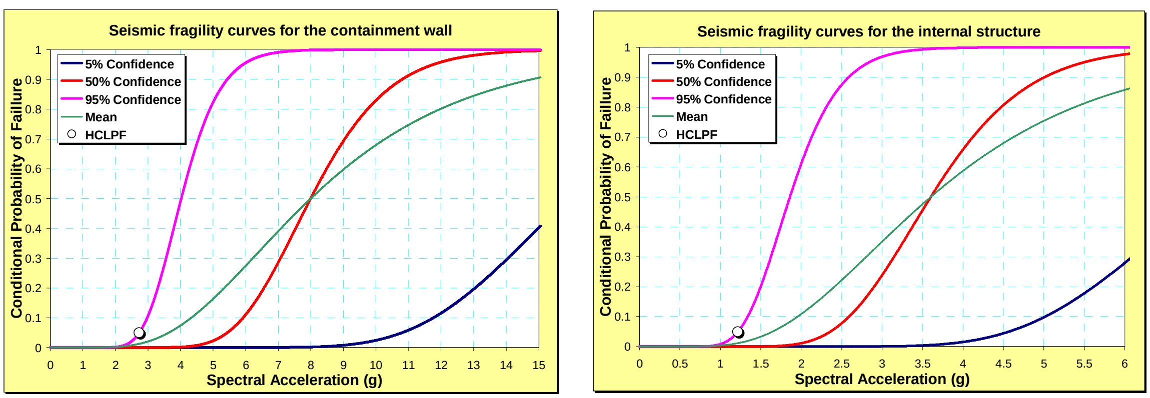
Figure 3. Critical zones for the reactor building.

For the internal structure, the modal spectral analyses results showed that the most critical element of this structure is the shear wall located at the axis 26 (see Figure 3.b). This shear wall has an opening of 10 feet width between the elevations 23'-6" and 50'-2". It is found that this part is the most critical for the entire shear wall. Therefore, the seismic fragility analyses of the internal structure are conducted for this shear wall at this location. The capacity in terms of HCLPF for this failure mode represents the lowest value among all potential failure modes.

The calculation details of the high confidence of low probability of failure (HCLPF) for the internal structure are presented in Hydro-Quebec (2010-b). However, the synthesis of the results is given in Table 3.1, and the fragility curves corresponding to 95%, median (50%), mean and 5% confidence levels are shown in Figure 4.b.

Table 3.1 HCLPF for the containment wall (CW) and the internal structure (IS).

			β_R		β_U	
	CW	**IS**	**CW**	**IS**	**CW**	**IS**
Strength factor (F_S)	**11.725**	**4.347**	**0**	**0**	**0.248**	**0.21**
Inelastic energy absorption factor (F_μ)	**2.019**	**2.094**	**0.09**	**0.082**	**0.262**	**0.263**
Structural response factor (F_{SR})	**1.156**	**1.354**	**0.217**	**0.244**	**0.209**	**0.219**
F_{SA}	**1**	**1**	**0.12**	**0.12**	**0**	**0**
F_{GMI}	**1**	**1**	**0**	**0**	**0**	**0**
F_δ	**1.156**	**1.354**	**0**	**0**	**0.145**	**0.159**
F_{Md}	**1**	**1**	**0**	**0**	**0.151**	**0.151**
F_{MC}	**1**	**1**	**0.1**	**0.15**	**0**	**0**
F_{EC}	**1**	**1**	**0.15**	**0.15**	**0**	**0**
F_{SSI}	**1**	**1**	**0**	**0**	**0**	**0**
Global safety factor	**27.37**	**12.325**	**0.235**	**0.257**	**0.418**	**0.402**
Median capacity in acceleration (A_M (*g*))	**CW**			**IS**		
	7.995			**3.602**		
***HCLPF* (*g*)**	**CW**			**IS**		
	2.726			**1.215**		



a. b.

Figure 4. Seismic fragility curves for the containment wall and the internal structure.

4. CONCLUSION

In this paper, the methodology adopted for the seismic fragility analyses of a CANDU 6 reactor building is presented. The fragility analyses are mainly based on the Electric Power Research Institute (EPRI) approach combined with structural models calibrated with ambient vibrations measurements results. The results are presented for the containment wall and for the internal structure.

Within the probabilistic seismic safety assessment, theses seismic fragility analyses are an important step in the evaluation of the seismic margins with regard to the capacity of components (structures) of the plant to sustain the new seismic demand.

The fragility analyses methodology adopted for this CANDU 6 NPP is particularly useful to engineers involved in fragility analyses of complex structural systems.

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